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SYNOPSIS: A new practical method for evaluation of soil liquefaction by using the Cone Penetration Test (CPT) was proposed. The critical normalized cone resistance, $(q_{c1})_{cr}$ was presented in terms of cyclic stress ratio, τ/σ_0' , and mean grain size of soils, D_{50}. Based on our proposed CPT-based liquefaction assessing correlation, we presented the typical examples of the use and application of CPT to assess liquefaction potential of past earthquake as well as to verify the effectiveness of soil improvement countermeasured to increase liquefaction resistance of soil.

INTRODUCTION

In general, there are two basic methods for evaluating the liquefaction potential of saturated sand deposits: (1) comparison of analytical stress or strain conditions induced by an earthquake with the stress or strain conditions that cause soil liquefaction measured by laboratory tests; and (2) evaluation based on the field performance of sand deposits in the area of previous earthquakes. When the latter method is used, the estimation of field liquefaction characteristics are determined either by use of empirical correlations of in-situ tests or by means of laboratory tests on undisturbed samples of soil deposits.

Recently, there has been growing interest in the use of empirical correlations between field liquefaction characteristics and the in-situ tests as an index of soil liquefaction resistance. Most of these are based on the Standard Penetration Test (SPT) (Seed & Idriss, 1971; Tatsuoka et al., 1980; Tokimatsu & Yoshimi, 1983; and Seed et al., 1983, 1985). The Cone Penetration Test (CPT) due to its simplicity, repeatability, accuracy and continuous record, has become more popular (Kokusho, 1987). The CPT-based liquefaction assessment techniques have been diveloped by several investigators (Zhou, 1980, 1981; Seed et al., 1983; Ishihara, 1985; Robertson & Campanella, 1985; Seed & DeAlba, 1986; and Shibata & Teparaksa, 1988).

In this paper, the proposed CPT-based liquefaction assessment method (Shibata and Teparaksa 1988) is reviewed. The application of CPT-base liquefaction assessment method in evaluating liquefaction potential of past earthquakes as well as in verification of the effectiveness of soil improvement countermeasures in the past is compared with field performance.

CPT-BASED LIQUEFACTION ASSESSMENT METHOD

Shibata and Teparaksa (1988) have proposed a new practical liquefaction assessment method by CPT for clean sands and fine soils based predominantly on the field performance that has occured during the past several earthquakes. Fig.1 shows the proposed correlation in the form of the normalized cone resistance, q_{c1}, and cyclic stress ratio, τ/σ_0' , that develops in the field during an earthquake for both clean sands with D_{50}>0.25mm (Fig.1(a)) and fine soils with D_{50}<0.25mm (Fig.1(b)). The black circles represent liquefaction condition while the white circles represent non liquefaction condition. The correlation derivative approaches were based on a review of case histories of soil liquefaction by the following procedures:

(1) The normalized cone resistance which is the corrected q_c-values for an effective overburden pressure, σ_0' of 1 kgf/cm^2 was approximated by

$$q_{c1} = C_1 \cdot q_c = \left(\frac{1.7}{\sigma_0'+0.7}\right) q_c \quad (\mathrm{kgf/cm^2}) \tag{1}$$

where C_1 is a function of σ_0' in kgf/cm^2 at the time and depth at which the CPT was performed.

(2) The cyclic stress ratio, τ/σ_0' , that develops in the field during an earthquake was estimated from the following equation (Tokimatsu & Yoshimi, 1983):

$$\frac{\tau}{\sigma_0'} = 0.1(M-1)\frac{\alpha_{max}}{g}\frac{\sigma_0}{\sigma_0'}(1-0.015Z) \tag{2}$$

where M is the earthquake magnitude, α_{max} is the maximum acceleration at ground surface, σ_0 and σ_0' are the total and effective overburden pressures and Z is the depth in meters.

(3) The data represented by the circle symbols in Fig.1 were based on field CPT investigation, while the triangle were based on the q_c-values converted from SPT data with the aid of the q_c/N-D_{50} correlation proposed by Robertson et al (1983).

(4) The critical boundary that separates the liquefiable from the non-liquefiable condition was expressed in terms of normalized cone resistance, cyclic stress ratio and mean grain size of soils as shown in Fig.1 by the functions:

$$(q_{c1})_{cr} = C_2\left[50+200\left\{\frac{(\tau/\sigma_0')-0.1}{(\tau/\sigma_0')+0.1}\right\}\right] \tag{3-a}$$

or

$$\left(\frac{\tau}{\sigma_0'}\right)_l = 0.1+0.2\left\{\frac{(q_{c1}/C_2)-50}{250-(q_{c1}/C_2)}\right\} \tag{3-b}$$

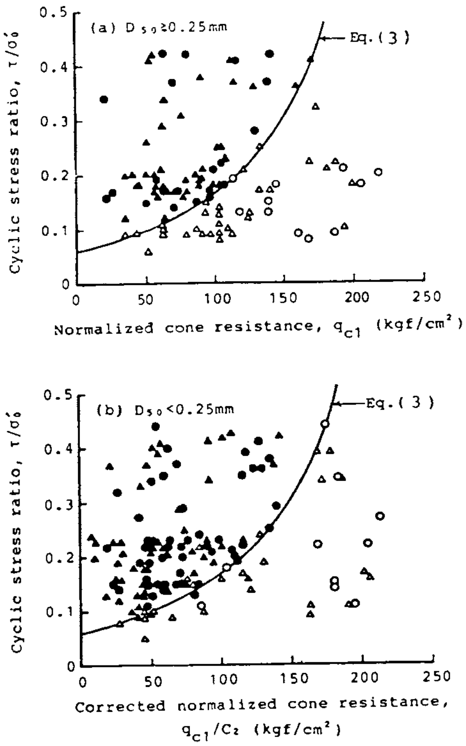

Fig. 1 Correlation between field liquefaction behaviour and normalized cone resistance

where C_2 is the correction factor: C_2=1.0 for clean sand with D_{50}>0.25mm and $C_2=D_{50}$(mm)/0.25(mm) for fine grained soils with D_{50}<0.25 mm. In Eq.(3-a), the $(q_{c1})_{cr}$ values can be estimated for any given values of cyclic stress ratio generated by the earthquake.

By the same manner, for any given field CPT q_c-value, the cyclic strength of soils can be evaluated from Eq.(3-b).

APPLICATION OF THE PROPOSED CPT-BASED CORRELATION

As aforementioned, the critical normalized cone resistance was expressed in terms of cyclic stress ratio and mean grain size. In this section, based on the field performance of past earthquakes, the liquefaction potential is evaluated in terms of critical cone resistance with the aid of Eq.(1), (2) and (3-a). Even few CPT studies have been reported for sites whose performance during past earthquakes has been known, however, several valuable field data in various areas and earthquakes will be discussed.

Assessment of Liquefaction Potentials

The application of CPT q_c-based method in assessing liquefaction potential are shown in Figure 2. The CPT data were obtained from two sites; A, at which liquefaction occured and B, at which there was no liquefaction during the 1983 Nihonkai-Chubu earthquake of magnitude 7.7 (Nagai & Suzuki, 1984). Site A was considered by Nagai & Suzuki to have liquefied but not site B.

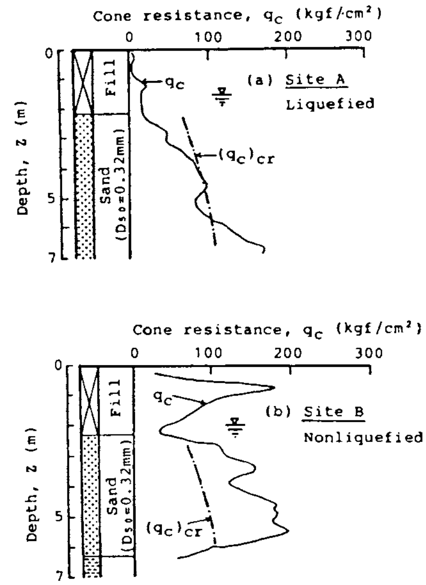

Fig. 2 Comparison of measured CPT q_c value with estimated critical $(q_c)_{cr}$ value in 1983 NihonKai-Chubu earthquake (Data from Nagai and Suzuki, 1984)

The possibility of liquefaction was evaluated by the proposed CPT-based method for sand layer immediately beneath artificial fills. The resulting critical cone resistances, $(q_c)_{cr}$, are plotted in Fig.2(a) and 2(b). This figure indicates that the upper 4 m of sand layer at site A is susceptible to liquefaction, because the measured q_c values are lower than the estimated critical $(q_c)_{cr}$ values, and that the 4-m thick sand layer at site B is resistant to liquefaction. Both predictions agree with observed field performance.

Another application of liquefaction potential assessing is shown in Figure 3. The CPT data was obtained from the site located in Victoria Town, Mexico. This site was liquefied during the Maxicali Earthquake in 1980 in which the earthquake magnitude is 6.7 and the epicenter was about 11 km from the Victoria Town. The great number of sand boils was formed at the ground surface in the form of small craters of silty fine sand. The maximum horizontal acceleration at ground surface was estimated from $\alpha_{max}=18.4\times10^{(0.302M)}\cdot D^{-0.8}$ which was about 148 gal or 0.15 g.

The possibility of liquefaction potential was assessed based on our CPT-based method for silty sand layer at about 3-5 m. deep. The critical cone resistances were plotted together with CPT data in Figure 3. The result indicates that the silty sand layer beneath fine sandy silt layer is susceptible to liquefy since the measured CPT q_c-values are lower than the estimated critical CPT data. The predictions agree with the obserbed field performance.

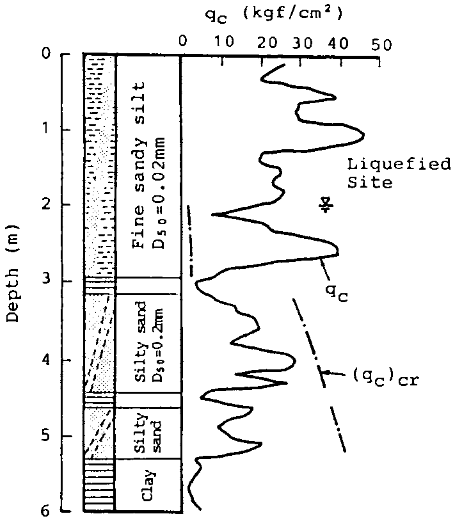

Fig. 3 Comparison of measured CPT q_c-value with estimated critical $(q_c)_{cr}$ value in 1980 Maxicali Earthquake (Data from Diaz-Rodriguez, 1984)

Verification of the Effectiveness of Soil Improvement Countermeasures

The damages of engineering structures, for instance, the settlement and tilting of structures resulting from the liquefaction of sand deposits during earthquake had been observed in the past earthquakes (the Niigata earthquake, 1964; Miyagiken-oki earthquake, 1978; and so on). These experiences push the geotechnical engineering profession to consider and prevent such damages whenever, the structures are constructed in the areas of liquefaction-susceptible deposits. In order to control and prevent the damages due to soil liquefaction, the soil improvement countermeasure techniques such as those summarized by Iwasaki (1986); vibrofloatation, dynamic compaction, pile, etc, are adopted, according to the soil conditions. This section presents the practical methods to verify the effectiveness of soil improvement countermeasure techniques against soil liquefaction based on our proposed CPT-based liquefaction assessment correlations.

Soil Improvement During Nihonkai-Chubu Earthquake (1983)

During the Nihonkai-chubu earthquake of 1983 with magnitude of 7.7 and maximum horizontal acceleration of 0.23g, an extensive damage to the road embankment due to the liquefaction of the ground was observed along the Nishirominami bypass in Aomori prefecture (Obinata, et al., 1985). After such damage, the embankments were repaired inwhich the vulnerable foundations of road embankment were stabilized by vibro-tamper soil improvement method.

Fig.4(a) shows the soil profiles and cone resistances before soil treatment at a failure section along the Nishirominami bypass, whereas the cone resistances after soil treatment are in Fig.4(b). Soil condition at this site was clean sands having D_{50}=0.27mm. The critical cone resistances estimated based on our CPT-based liquefaction assessment method are also included in Fig.4. It can be seen that for before compaction case, liquefaction occurs because the measured cone resistances are smaller than critical CPT resistances. On the other hand, if earthquake like the Nihonkai-chubu earthquake again occurs, the ground after soil treatment is unvulnerable to liquefaction.

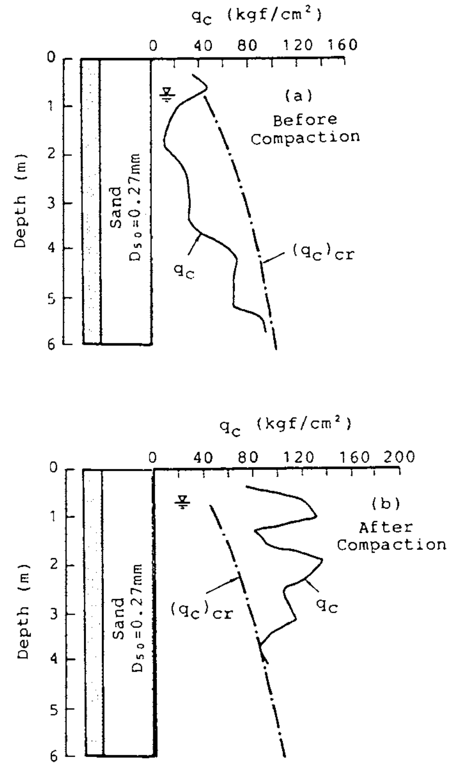

Fig. 4 Verification of the effectiveness of countermeasures against soil liquefaction based on CPT-based liquefaction method (Data from Obinata, et,al. 1985)

Another example site during the Nihonkai-chubu earthquake of 1983 is at Hashiro-Gata. The earthquake Magnitude of 7.7 and maximum horizontal acceleration of 165.5 gals was caused the damage to reclaimation dike in Hashiro-Gata. After such damage, the most economic method to prevent liquefaction by increasing effective stress in sand layer to upgrade its strength against liquefaction was adopted (Yamamura et, al., 1987).

Field observation was conducted to confirm the soil improvement countermeasures of an increasing effective stress in sand layer by means of Cone Penetration Test as shown in Figure 5. Figure 5(a) shows the soil profile and cone resistance before improvement at the failure section of reclaimed dike and Figure 5(b) was the after improvement case. The critical cone resistances based on our CPT-based method are also included. It is clear that liquefaction accurs for before improvement case since the measured cone resistances are smaller than critical CPT resistances. On the other hand, for after improvement case the ground is safe for liquefaction.

Soil Improvement in Rostock, GDR

The Vibro Wing method for deep compaction of natural and dredged cohesionless materials was adopted as a soil improvement method during the construction of a large harbour of general cargo in Rostock, GDR. The purpose of this soil improvement technique is to prevent the damage due to liquefaction from earthquake of magmitude 6.5 and maximum horizontal acceleration of 0.2 g. The soil condition at this site consists of top sand fill about 5 m.

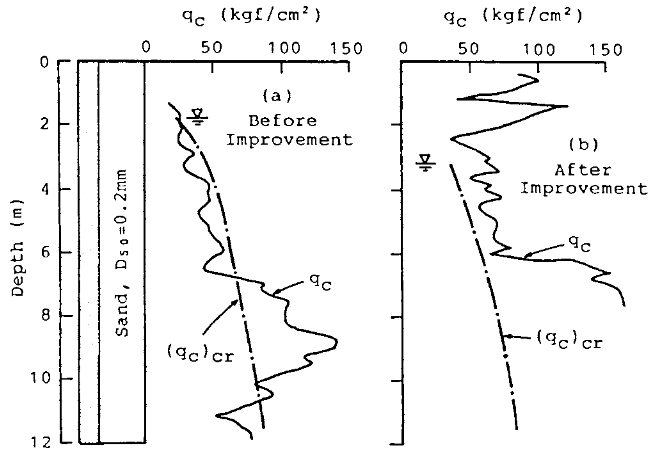

Fig. 5 Verification of the effectiveness of countermeasures against liquefaction at Reclaimed dikes in Hashiro-Gata (Data from Yamamura, et.al. 2987)

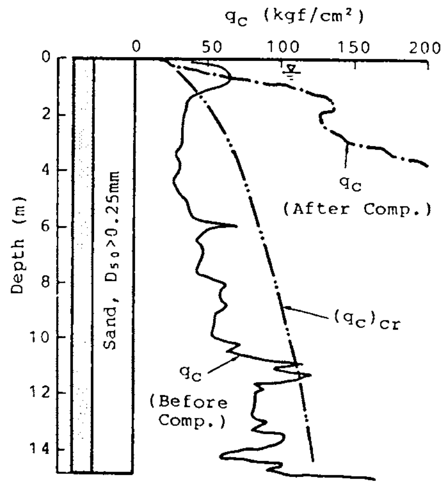

Fig. 6 Verification of the effectiveness of soil treatment against liquefaction at Rostock, GDR (Data from Massarsch & Lindberg, 1984)

deep on the original ground of clean sand with D_{50}>0.25mm. (Massarsch and Lindberg, 1984). Figure 6 shows the results of cone penetration test both before and after soil improvement as well as the estimated critical cone resistance, $(q_c)_{cr}$. The result shows that for before compaction case, the ground is vulnurable to liquefaction whereas for after compaction case, the ground is unvulnurable to liquefaction.

Soil Improvement Countermeasures in Florida and Hampton, USA

The examples of an evaluating of effectiveness of soil improvement countermeasures in Florida and Hampton, USA were presented in Figure 7 and 8, respectively. Figure 7(a) shows the CPT resistance of the ground which is the mine tailing sand (Schmertmann, 1978). During mining, the soils was removed about 5 m. deep and fine sand was replaced by hydraulically fill. This mine was stoped about 20 year ago, the soil condition is rather loose. Figure 7 (b) shows the result of CPT test at the site after improving by vibratory roller with 10.8 ton weight. The critical cone resistances, $(q_c)_{cr}$, were also included in Figure 7, which was estimated based on the earthquake magnitude of 5.75 with maximum acceleration of 0.16 g. This earthquake information was assumed from the seismic probability map of U.S.A. The result shows that for before compaction case it is susceptable to liquefaction whereas after compaction is unsusceptable to liquefaction.

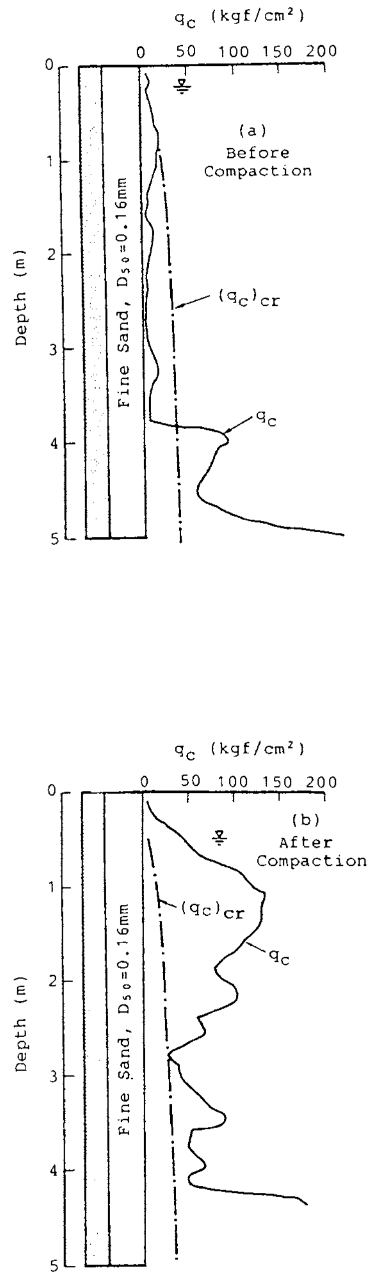

Fig. 7 verification of the effectiveness of soil improvement against liquefaction in Florida (Data from Schmertman, 1978)

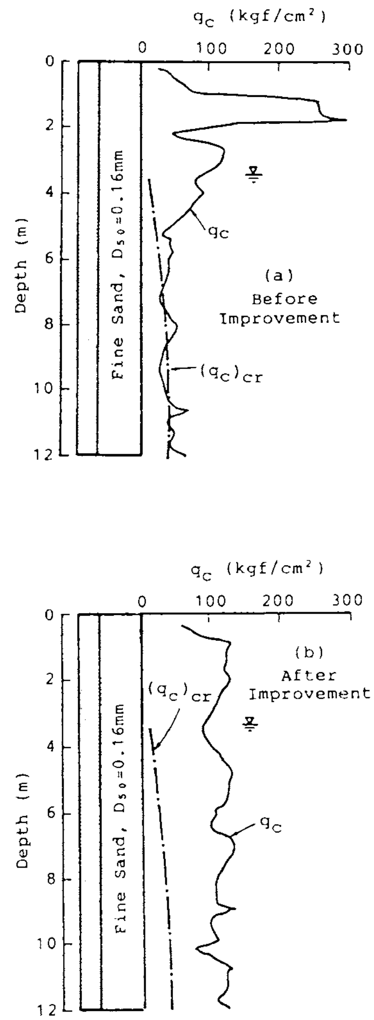

Fig. 8 Verification of the effectiveness of counter-measures against liquefaction in Hampton (Data from Brown, 1977)

Figure 8 shows the CPT testing results at the site in Hampton, USA inwhich Figure 8(a) is the test of original ground condition before improvement and Figure 8(b) is after improvement case. The soil condition consists of fine sand with D_{50}=0.16mm. The vibroflotation soil improvement technique was adopted as the method to increase liquefaction resistance of soils. (Brown, 1978). The critical cone resistances, $(q_c)_{cr}$, estimated based on the seismic probability map of U.S.A. of earthquake magnitude of 5.75 with maximum acceleration of 0.16 g were also included in Figure 8. The verification of an effectiveness of soil improvement technique by our proposed CPT-based method indicated that after improvement, it is unvulnerable to liquefaction whereas before improvement, the ground is vulnerable to liquefaction.

CONCLUSION

The proposed CPT-based liquefaction assessing method (Shibata and Teparaksa, 1988) in terms of cyclic stress ratio, τ/σ_0' , and mean grain size, D_{50}, (Figure 1) was reviewed. This correlation are based on the field performance that has occured during the past several earthquakes. The use of this method is convenient, since it can avoid the difficulties concerning undisturbed sampling and laboratory testing by making the most direct use of the field experiences.

The application of CPT-based correlation in evaluating liquefaction potential of soils compared to field evidence of Nihonkai-Chubu earthquake (1983) was in good agreement. The verification of effectiveness of soil improvement countermeasures of various soil improvement technique based on our CPT correlation was presented.

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